CHAPTER 500
TRAFFIC INTERCHANGES

Topic 501 - General

Index 501.1 - Concepts

A traffic interchange is a combination of ramps and grade separations at the junction of two or more highways for the purpose of reducing or eliminating traffic conflicts, to improve safety, and increase traffic capacity. Crossing conflicts are eliminated by grade separations. Turning conflicts are either eliminated or minimized, depending upon the type of interchange design.

501.2 Warrants

All connections to freeways are by traffic interchanges. An interchange or separation may be warranted as part of an expressway (or in special cases at the junction of two non-access controlled highways), to improve safety or eliminate a bottleneck, or where topography does not lend itself to the construction of an intersection.

501.3 Spacing

The minimum interchange spacing shall be 1.5 km in urban areas, 3.0 km in rural areas, and 3.0 km between freeway-to-freeway interchanges and local street interchanges. To improve operations of closely spaced interchanges the use of auxiliary lanes, grade separated ramps, collector distributor roads, and/or ramp metering may be warranted.

See Design Information Bulletin No. 77 for additional information on interchange spacing, including the procedural and documentation requirements to be fulfilled prior to requesting an exception to the above standards.

Topic 502 - Interchange Types

502.1 General

The selection of an interchange type and its design are influenced by many factors including the following: the speed, volume, and composition of traffic to be served, the number of intersecting legs, the standards and arrangement of the local street system including traffic control devices, topography, right of way controls, local planning, proximity of adjacent interchanges, community impact, and cost. Even though interchanges are, of necessity, designed to fit specific conditions and controls, it is desirable that the pattern of interchange ramps along a freeway follow some degree of consistency. It is frequently desirable to rearrange portions of the local street system in connection with freeway construction in order to effect the most desirable overall plan of traffic service and community development.

Interchange types are characterized by the basic shapes of ramps: namely, diamond, loop, directional, hook, or variations of these types. Many interchange designs are combinations of these basic types. Schematic interchange patterns are illustrated in Figure 502.2 and Figure 502.3. These are classified as: (a) Local street interchanges and (b) Freeway-to-freeway interchanges. See Chapter X of "A Policy on Geometric Design of Highways and Streets," AASHTO, 1994, for additional examples.

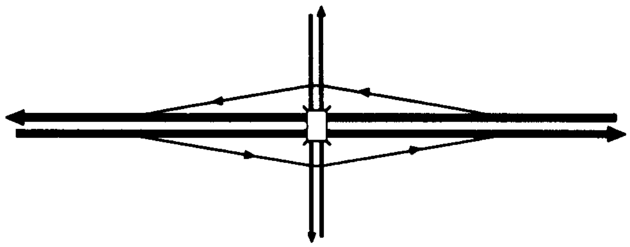
502.2 Local Street Interchanges

<u>The use of isolated off ramps or partial interchanges should be avoided because of the potential for wrong-way movements and added driver confusion.</u> In general, interchanges with all ramps connecting with a single cross street are preferred.

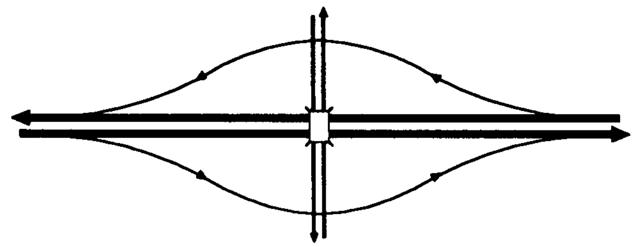
(a) Diamond Interchange--The simplest form of interchange is the diamond. Diamond interchanges provide a high standard of ramp alignment, direct turning maneuvers at the crossroads, and usually have minimum construction costs. The diamond type is adaptable to a wide range of traffic volumes. The capacity is limited by the capacity of the intersection of the ramps at the crossroad. This capacity may be increased by widening the ramps to two or three lanes at the crossroad and by widening the crossroad in the intersection area. Crossroad widening will increase the length of undercrossings and the width of overcrossings, thus adding to the bridge cost. Ramp intersection capacity analysis is discussed in Topic 406.

Figure 502.2

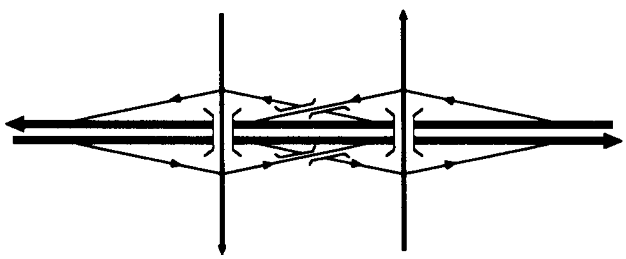
Typical Local Street Interchanges



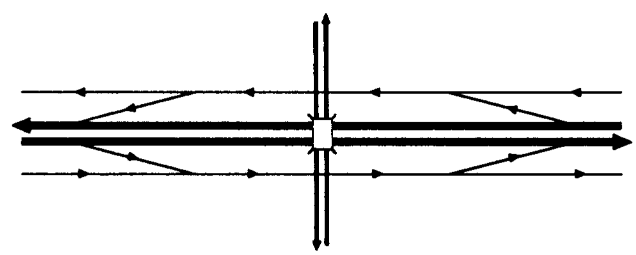
TYPE L-1



TYPE L-2



TYPE L-3



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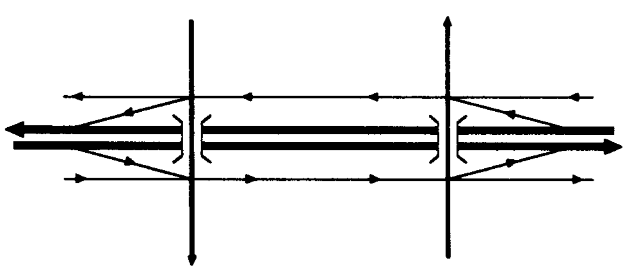
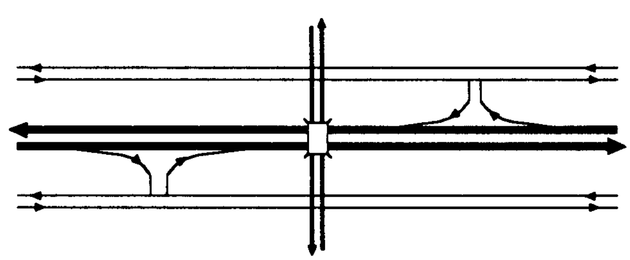


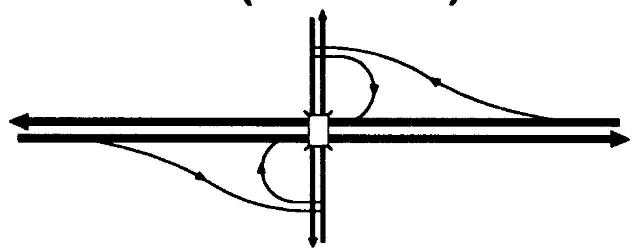
TYPE L-5



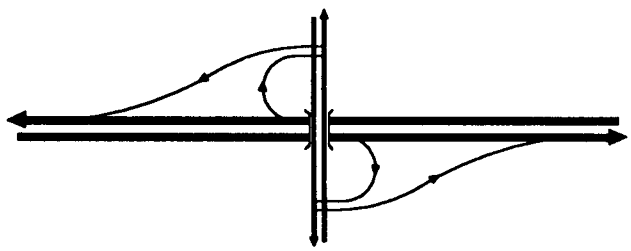
TYPE L-6

Figure 502.2

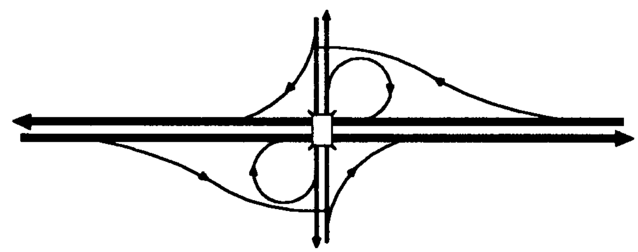
Typical Local Street Interchanges (continued)



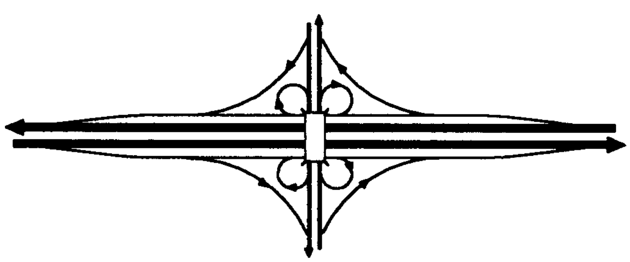
TYPE L-7



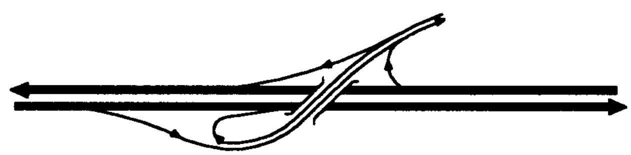
TYPE L-8



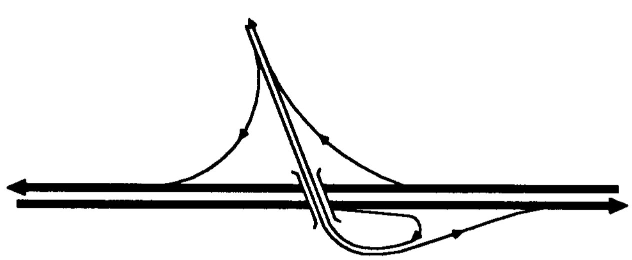
TYPE L-9



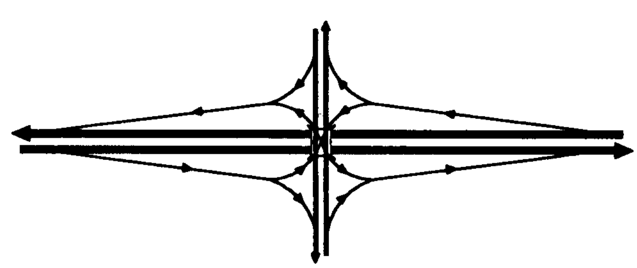
TYPE L-10



TYPE L-11



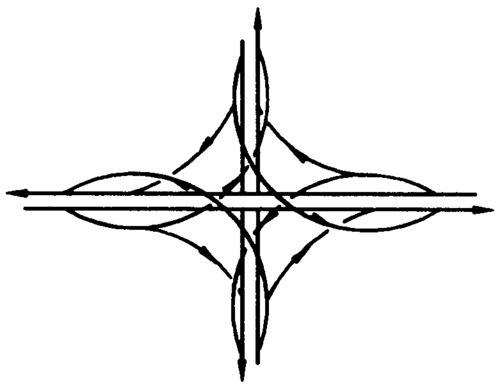
TYPE L-12



TYPE L-13

Figure 502.3

Typical Freeway-to-freeway Interchanges



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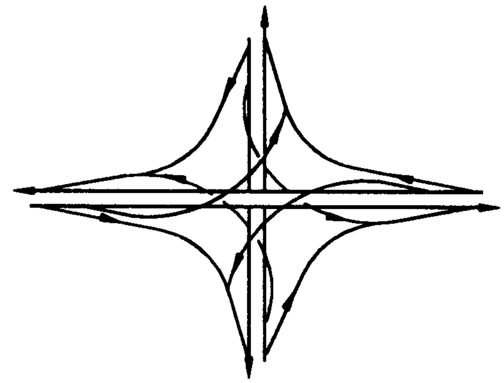
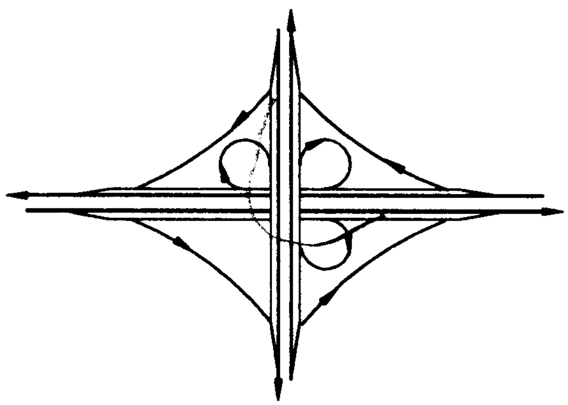
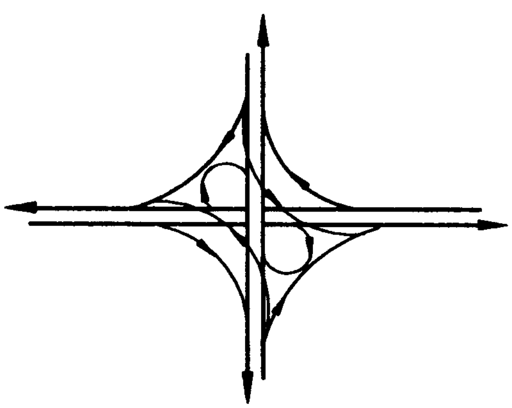


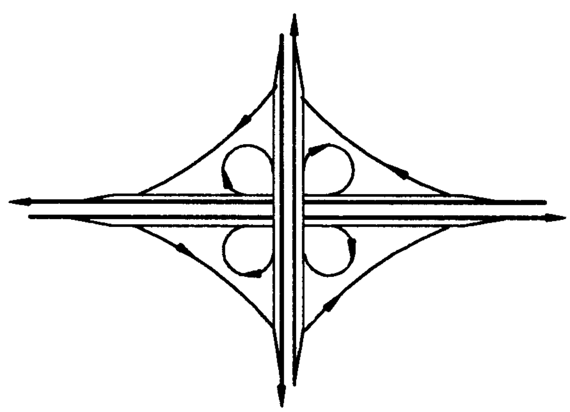
TYPE F-1 (ALT "B")



TYPE F-2



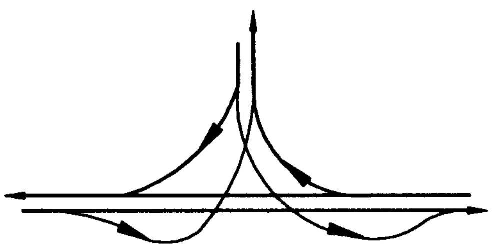
TYPE F-3



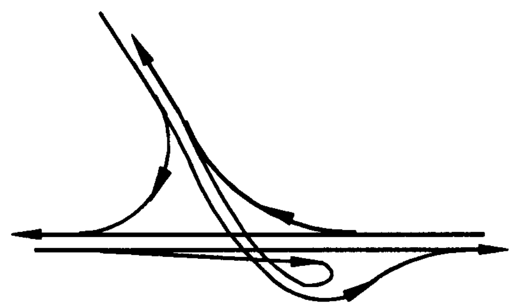
TYPE F-4

Figure 502.3

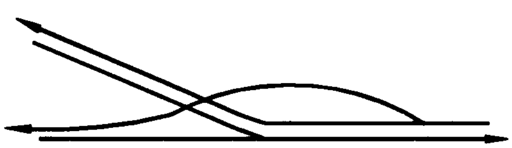
Typical Freeway-to-freeway Interchanges (continued)



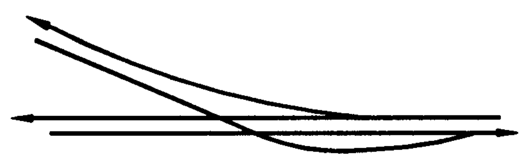
TYPE F-5



TYPE F-6



TYPE F-7



TYPE F-8

The compact diamond (Type L-1) is most adaptable where the freeway is depressed or elevated and the cross street retains a straight profile. Type L-1's are suitable where physical, geometric or right of way restrictions do not permit a spread diamond configuration.

The spread diamond (Type L-2) is adaptable where the grade of the cross street is changed to pass over or under the freeway. The ramp terminals are spread in order to achieve maximum sight distance and minimum intersection cross slope, commensurate with construction and right of way costs, travel distance, and general appearance. A spread diamond has the advantage of flatter ramp grades, greater crossroads left-turn storage capacity, and the flexibility of permitting the construction of future loop ramps if required.

The split diamond with braids (Type L-3) may be appropriate where two major crossroads are closely spaced.

(b) Interchanges with Parallel Street Systems--Types L-4, L-5 and L-6 are interchange systems used where the freeway alignment is placed between parallel streets. Types L-4 and L-5 are used where the parallel streets will operate with one-way traffic. In Type L-4 slip ramps merge with the frontage street and in Type L-5 the ramps terminate at the intersection of the frontage road with the cross street, forming five-legged intersections. In Type L-6 the freeway ramps connect with two-way parallel streets. The parallel streets in the Types L-4, L-5 and L-6 situation are usually too close to the freeway to permit ramp intersections on the cross street between the parallel frontage streets.

The "hook" ramps of the Type L-6 are often forced into tight situations that lead to less than desirable geometrics. The radius of the curve at the approach to the intersection should exceed 50 m and a tangent of at least 50 m should be provided between the last curve on the ramp and the ramp terminal. Special attention should always be given to exit ramps that end in a hook to ensure that adequate sight distance around the curve, deceleration prior to the curve or end of anticipated queue, and adequate superelevation for anticipated driving speeds can be developed.

(c) Cloverleaf Interchanges--The simplest cloverleaf interchange is the two-quadrant cloverleaf, Type L-7 or Type L-8, or a combination where the two loops are on the same side of the cross street. Type L-7 eliminates the need for left-turn storage lanes, on or under the structure, thus reducing the structure costs. These interchanges should be used only in connection with controls which preclude the use of diamond ramps in all four quadrants. These controls include right of way controls, a railroad track paralleling the cross street, and a short weaving distance to the next interchange.

The Type L-9, partial cloverleaf interchange, provides loop on-ramps in addition to the four diamond-type ramps. This interchange is suitable for large volume turning movements. Left-turn movements from the crossroads are eliminated, thereby permitting two-phase operation at the ramp intersections when signalized. Because of this feature, the Type L-9 interchange usually has capacity to handle the volume of interchange traffic which can be accommodated on the crossroads.

The four-quadrant cloverleaf interchange (Type L-10) has free-flow characteristics for all movements. It has the disadvantage of a higher cost than a diamond or partial cloverleaf design and a relatively short weaving section between the loop ramps which limits capacity. Collector-distributor roads should be incorporated in the design of four-quadrant cloverleaf interchanges to separate the weaving conflicts from the through freeway traffic.

(d) Trumpet Interchanges--A trumpet design, Type L-11 or L-12, may be used when a crossroads terminates at a freeway. This design should not be used if future extension of the crossroads is probable. The diamond interchange is preferable if future extension of the crossroads is expected.

(e) Single Point Interchange (SPI)--The Type L-13 is a concept which essentially combines two separate diamond ramp intersections into one large at-grade intersection. It is also known as an urban interchange. Type L-13 requires approximately the same right-of-way as the compact diamond. However, the construction cost is substantially higher due to the structure requirements. The capacity of the L-13 can substantially exceed that of a compact diamond if long signal times can be provided and left turning volumes are balanced.

This additional capacity may be offset if nearby intersection queues interfere with weaving and storage between intersections. The disadvantages of the L-13 are: 1) Future expansion of the interchange is extremely difficult; 2) Stage construction for retrofit situations is costly; 3) Long structure spans require higher than normal profiles and deeper structure depths; and 4) Poor pedestrian circulation that could lead to providing a grade separated pedestrian structure if pedestrian volumes are high.

Typically, the SPI is best suited for undercrossings; it is difficult to provide good geometrics where the cross street rolls up and over the freeway.

Special attention should be given to signing, striping, lighting and sight distance due to driver unfamiliarity with the non-conventional geometrics and features of the L-13. The SPI is significantly longer and wider than conventional intersections, therefore, pavement delineation is particularly important in order to guide the driver through the intersection. The sight distance to pavement delineation on crest vertical curves is reduced as compared to flat grades or sag vertical curves. (Night-time sight distance on sag vertical curves can be supplemented by street lighting.) Proposals for Type L-13 interchanges on crest vertical curves should be discussed with the Project Development Coordinator and Traffic Operations Reviewer early in the project development process.

Designers should refer to the 1994 AASHTO Publication "A Policy on Geometric Design of Highways and Streets" (the Green Book) for more detailed information on Single Point Interchanges.

502.3 Freeway-to-freeway Interchanges

(1) General. The function of the freeway-to-freeway interchange is to link freeway segments together so as to provide the optimum highway system. Parameters such as cost, environment, community values, traffic volumes, route continuity, map relatability, and safety should all be considered. Both the sign route and the major traffic volume should be to the left at a freeway-to-freeway interchange, if possible.

(2) Design Considerations.

(a) Cost--The differential cost between interchange types is often significant. A cost-effective approach will tend to assure that an interchange is neither over nor underdesigned. Decisions as to the relative values of the previously mentioned parameters must be consistent with decisions reached on adjacent main line freeways.

(b) System Balance--The freeway-to-freeway interchange is a critical link in the total freeway system. The level of traffic service provided will have impact upon the mobility and overall effectiveness of the entire roadway system. For instance, traffic patterns will adjust to avoid repetitive bottlenecks, and to the greatest degree possible, to temporary closures, accidents, etc. The freeway-to-freeway interchange should provide flexibility to respond to these needs so as to maximize the cost effectiveness of the total system.

(c) Elimination of Connections--Freeway-to-freeway interchanges need not include all possible turning movements.

Connections serving minor traffic volumes or significantly out-of-direction traffic movements should be omitted unless it can be demonstrated that traffic service and other benefits justify the costs. Considerations include:

- Traffic volumes--Turning traffic volumes may be nominal or a small percentage of the total interchanging traffic.

- Circuitry--Connections may only serve significantly out-of-direction traffic movements.

- Freeway location--Where three freeways cross so as to form a relatively small triangle, the omission of the backward freeway-to-freeway connections from one leg of the triangle to another may have little negative impact on local or through traffic service.

- Use of local streets--Low turning volumes may be accommodated reasonably well by way of local interchanges and the local street system. There may be both traffic operational advantages and economic savings from utilizing and improving this local system in lieu of providing the freeway-to-freeway connections.

- Staging--Staging possibilities should be thoroughly assessed. Provisions should be made for adding or upgrading ramps and connectors at a later time. For example, an initial loop ramp might be later upgraded to direct connector.

- Effect on other traffic movements--Provision of minor movements may be detrimental to traffic operation on major branch connections and the main line freeways.

- Costs--All construction and right of way savings and costs attributable to the elimination of turning movements should be considered. This includes possible additional local interchange and street costs as well as reductions in the freeway-to-freeway interchange costs.

- Signing--Freeway-to-freeway traffic may be signed via the local street system. Routes should be sufficiently direct and well oriented to insure that the unfamiliar driver can follow them.

(d) Local Traffic Service--In metropolitan areas a freeway-to-freeway interchange is usually superimposed over an existing street system. Local and through traffic requirements are often in conflict. Combinations of local and freeway-to-freeway interchanges can result in designs that are both costly and so complex that the important design concepts of simplicity and consistency are compromised. Therefore, alternate plans separating local and freeway-to-freeway interchanges should be fully explored. Less than desirable local interchange spacing may result; however, this may be compensated for by upgrading the adjacent local interchanges and street system.

Local traffic service interchanges should not be located within freeway-to-freeway interchanges unless geometric standards and level of service will be substantially maintained.

(e) Alignment--It is not considered practical to establish fixed freeway-to-freeway interchange alignment standards. An interchange must be designed to fit into its environment. Alignment is often controlled by external factors such as terrain, buildings, street patterns, route adoptions, and community value considerations. Normally, loops have radii in the range of 50 m to 65 m and direct connections should have minimum radii of 260 m. Larger radii may be proper in situations where the skew or other site conditions will result in minimal increased costs. Direct connection radii of at least 350 m are desirable from a traffic operational standpoint. High alignment and sight distance standards should be provided where possible.

Drivers have been conditioned to expect a certain standard of excellence on California freeways. The designer's challenge is to provide the highest possible standards consistent with cost and level of service.

(3) *Types*. Several freeway-to-freeway interchange design configurations are shown on Figure 502.3. Many combinations and variations may be formed from these basic interchange types.

(a) Four-Level-Interchange--Direct connections are appropriate in lieu of loops when required by traffic demands or other specific site conditions. The Type F-1 interchange with all direct connections provides the maximum in mobility and safety. However, the high costs associated with this design require that the benefits be fully substantiated.

The Type F-1 Alternative "A" interchange utilizes a single divergence ramp for traffic bound for the other freeway; then provides a secondary directional split. Each entrance ramp on a Type F-1A interchange is provided separately. The advantages of the Type F-1A are: 1) reduced driver confusion since there is only one exit to the other freeway, and 2) operations at the entrance may be improved since the ramps merge with the mainline one at a time.

The Type F-1 Alternative "B" interchange provides separate directional exit ramps and then merges the entering traffic into a single ramp before converging with the mainline. Since the Type F-1B combines traffic from two ramps before entering the freeway, it is important to verify that adequate weaving capacity is provided beyond the entrance. Separating the directional split of exiting traffic reduces the volume to each of the two ramps and therefore may improve the level of service of the weave section prior to the exit.

Design for a four-level interchange may combine the configuration of the Type F1-A and F1-B interchange to best suit the conditions at a given location.

(b) Combination Interchanges--The three-quadrant cloverleaf, Type F-2, with one direct connection may be necessary where a single move carries too much traffic for a loop ramp or where the one quadrant is restricted by environmental, topographic, or right of way controls.

The two-loop, two-direct connection interchange, Type F-3, is often an appropriate solution. The weaving conflicts which ordinarily constitute the most restrictive traffic constraint are eliminated, yet cost and right of way requirements may be kept within reasonable bounds. Consideration should be given to providing an auxiliary lane in advance of the loop off-ramps to provide for vehicle deceleration.

(c) Four-Quadrant Cloverleaf--The four-quadrant cloverleaf with collector-distributor roads, Type F-4, is ordinarily the most economical freeway-to-freeway interchange solution when all turning movements are provided. The four-quadrant cloverleaf is generally applicable in situations where turning volumes are low enough to be accommodated in the short weaving sections. It should be designed with collector-distributor roads to separate weaving conflicts from the through freeway traffic.

(d) Freeway Terminal Junction--Types F-5, F-6, F-7, and F-8 are examples of interchange designs where one freeway terminates at the junction with another freeway. In general, the standard of alignment provided on the left or median lane connection from the terminating freeway should equal or approach as near as possible that of the terminating freeway. Terminating the median lane on a loop should be avoided. It is preferable that both the sign route and the major traffic volume be to the left at the branch connection diverge. The choice between Types F-7 and F-8 should include considerations of traffic volumes, route continuity, and map relatability. When these considerations are in conflict, the choice is made on the basis of judgment of their relative merits.

Topic 503 - Interchange Design Procedure

503.1 Basic Data

Data relative to community service, traffic, physical and economic factors, and potential area development which may materially affect design, should be obtained prior to interchange design. Specifically, the following information should be available:

(a) The location and standards of existing and proposed local streets including types of traffic control.

(b) Existing and proposed land use including such developments as shopping centers, recreational facilities, housing developments, schools, and other institutions.

(c) A traffic flow diagram showing average daily traffic and design hourly volumes, as well as time of day (a.m. or p.m.), anticipated on the freeway ramps and affected local streets or roads.

(d) The relationship with adjacent interchanges.

(e) The location of major utilities, railroads, or airports.

503.2 Approval

Interchanges are among the major design features which are to be reviewed by the Project Development Coordinator, other Headquarters staff, and the FHWA Transportation Engineer, as appropriate. Major design features include the freeway alignment, geometric cross section, location of separation structures, closing of local roads, frontage road construction, and work on local roads. Particularly close involvement should occur during preparation of the Project Study Report and Project Report (see the Project Development Procedures Manual). Such reviews can be particularly valuable when exceptions from advisory or mandatory design standards are being considered and alternatives are being sought.

Topic 504 - Interchange Design Standards

504.1 General

Topic 504 discusses the standards that pertain to both local service interchanges (various ramp configurations) and freeway-to-freeway connections. The design standards, policies and practices covered in Indexes 504.2, and 504.5 through 504.8 are typically common to both ramp and connector interchange types. Indexes 504.3 and 504.4 separately discuss ramp standards and freeway-to-freeway connector standards, respectively.

504.2 Freeway Entrances and Exits

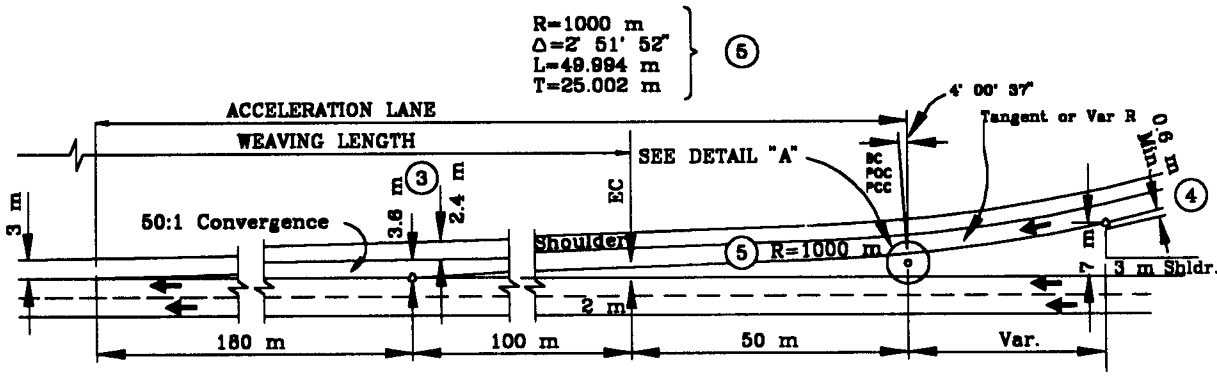
(1) *Basic Policy.* **All freeway entrances and exits, except for direct connections with median High Occupancy Vehicle lanes, shall connect to the right of through traffic.**

(2) *Standard Designs.* <u>Design of freeway entrances and exits should conform to the standard designs illustrated in Figure 504.2A (single lane), and Figure 504.3C (two lane entrances and exits) and/or Figure 504.4 (diverging branch connections), as appropriate.</u>

The minimum deceleration length shown on Figure 504.2A shall be provided prior to the first curve beyond the exit nose to assure adequate distance for vehicles to decelerate before entering the curve. <u>The same standard should apply for the first curve after the exit from a collector-distributor road.</u> The range of minimum "DL" (distance) vs. "R" (radius) is given in the table in the lower part of Figure 504.2A above the exit ramp diagram. Strong consideration should be given to lengthening the "DL" distance given in the table when the subsequent curve is a descending loop or hook ramp, or if the upstream condition is a sustained downgrade (see AASHTO, A Policy on Geometric Design of Highways and Streets, 1994, Chap. 10 for additional information).

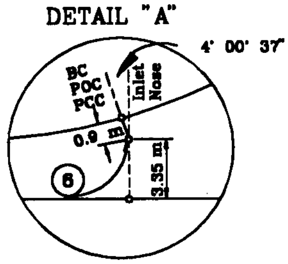
Figure 504.2A

Single Lane Freeway Entrances and Exits

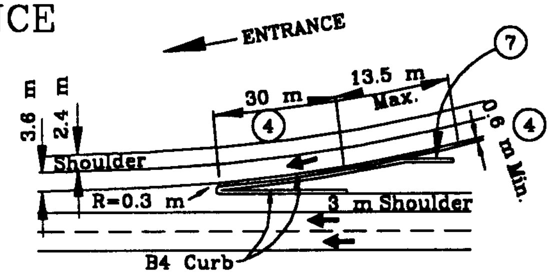


ENTRANCE

DETAIL "A"



INLET NOSE

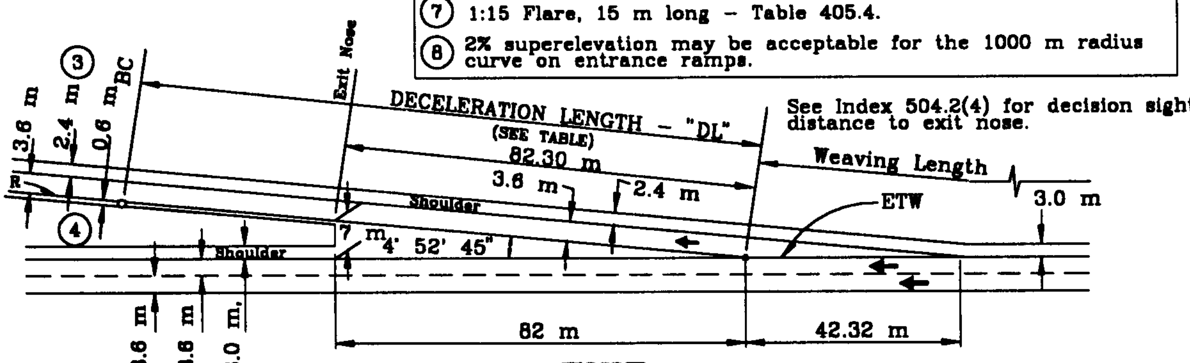


OPTIONAL B-4 CURB AT INLET NOSE
(See Index 504.3(10))

R (m)	Min. DL (m) ②
Less than 90	180
90–150	150
151–300	130
over 300	82.3

NOTES:

1. Minimum length between exit nose and end of ramp is 160 m for full stop at end of ramp.
2. "DL" distance should be lengthened for descending, short radius curves, or if entered from a sustained downgrade.
3. On single lane freeway to freeway connections the right paved shoulder shall be 3 m.
4. A 1.2 m left paved shoulder is preferred in urban areas; on freeway to freeway connections it shall be 1.5 m.
5. When freeway is not on tangent alignment, select radius to approximate same degree of convergence(see Index 504.2(3)).
6. Locate as if it were to be center of a 0.3 m radius curb nose.
7. 1:15 Flare, 15 m long – Table 405.4.
8. 2% superelevation may be acceptable for the 1000 m radius curve on entrance ramps.



EXIT

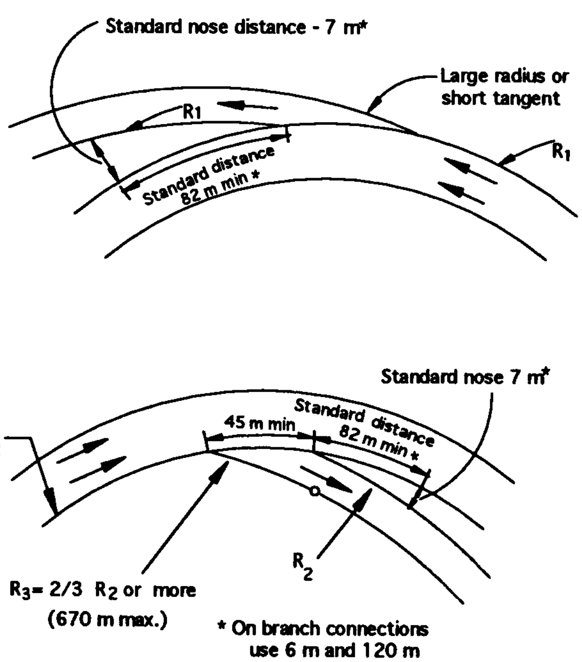
The exit nose shown on Figure 504.2A may be located downstream of the 7 m dimension; however, the maximum paved width between the mainline and ramp shoulder edges should be 6 m. Also, see pavement cross slope requirements in Index 504.2(5).

Refer to the HOV Guidelines and Ramp Meter Design Guidelines for additional information specific to direct connections to HOV lanes and metered entrance ramps and connectors.

(3) *Location on a Curve.* Freeway entrances and exits should be located on tangent sections wherever possible in order to provide maximum sight distance and optimum traffic operation. Where curve locations are necessary, the ramp entrance and exit tapers should be curved also. The radius of the exit taper should be about the same as the freeway edge of traveled way in order to develop the same degree of divergence as the standard design (see Figure 504.2B).

Figure 504.2B

Location of Freeway Ramps on a Curve



On entrance ramps the distance from the inlet nose (4.25-meter point) to the end of the acceleration lane taper should equal the sum of the distances shown on Figure 504.2A. The 50:1 taper may be curved to fit the conditions, and the 1000 m radius curve may be adjusted (see Figure 504.2A, note 5).

When an exit must be located where physical restrictions to visibility cannot be corrected by cut widening or object removal, an auxiliary lane in advance of the exit should be provided. The length of auxiliary lane should be a minimum 180 m, 300 m preferred.

(4) *Design Speed Considerations.* In the design of interchanges it is important to provide vertical and horizontal alignment standards which are consistent with driving conditions expected on branch connections. Sight distance on crest vertical curves should be consistent with expected approach speeds.

(a) Freeway Exit--The design speed at the exit nose should be 80 km/h or greater for both ramps and branch connections.

Decision sight distance given in Table 201.7 should be provided at freeway exits and branch connectors. At secondary exits on collector-distributor roads, a minimum of 190 m of decision sight distance should be provided. In all cases, sight distance is measured to the center of ramp lane right of the nose.

(b) Freeway Entrance--The design speed at the inlet nose should be consistent with approach alignment standards. If the approach is a branch connection or diamond ramp with high alignment standards, the design speed should be at least 80 km/h.

(c) Ramps--See Index 504.3(1)(a).

(d) Freeway-to-Freeway Connections -- See Index 504.4(2).

(5) *Grades.* Grades for freeway entrances and exits are controlled primarily by the requirements of sight distance. Ramp profile grades should not exceed 8% with the exception of descending entrance ramps and ascending exit ramps, where a 1% steeper grade is allowed. However, the 1% steeper grade should be avoided on

descending loops to minimize overdriving of the ramp (see Index 504.3 (7)).

Profile grade considerations are of particular concern through entrance and exit gore areas. In some instances the profile of the ramp or connector, or a combination of profile and cross slope, is sufficiently different than that of the freeway through lanes that grade breaks across the gore may become necessary. Where adjacent lanes or lanes and gore areas at freeway entrances and exits are not in the same plane, the algebraic difference in pavement cross slope should not exceed 5%.

In addition to the effects of terrain, grade lines are also controlled by structure clearances (see Indexes 204.6 and 309.2). Grade lines for overcrossing and undercrossing roadways should conform to the requirements of HDM Topic 104 Roads Under Other Jurisdictions.

(a) Freeway Exits--Vertical curves located just beyond the exit nose should be designed with a minimum 80 km/h stopping sight distance. Beyond this point, progressively lower design speeds may be used to accommodate loop ramps and other geometric features.

Ascending off-ramps should join the crossroads on a reasonably flat grade to expedite truck starts from a stopped condition. If the ramp ends in a crest vertical curve the last 15 m of the ramp should be on a 5% grade or less. There may be cases where a valley gutter is necessary to prevent crossroads water from draining onto the ramp.

On descending off-ramps, the sag vertical curve at the ramp terminal should be a minimum of 30 m in length.

(b) Freeway Entrances--Entrance profiles should approximately parallel the profile of the freeway for at least 30 m prior to the inlet nose to provide intervisibility in merging situations. The vertical curve at the inlet nose should be consistent with approach alignment standards.

Where truck volumes (three-axle or more) exceed 20 per hour on ascending entrance ramps to freeways and expressways with sustained upgrades exceeding 2%, a 450 m length of auxiliary lane should be provided in order to insure satisfactory operating conditions. Additional length may be warranted based on the thorough analysis of the site specific grades, traffic volumes, and calculated speeds; and after consultation with representatives of the Headquarters Traffic Operations Program and the Office of Project Planning and Design. Also, see Index 204.5 "Sustained Grades".

504.3 Ramps

(1) General.

(a) Design Speed -- When ramps terminate at an intersection at which all traffic is expected to make a turning movement, the minimum design speed along the ramp should be 40 km/h. When a “through” movement is provided at the ramp terminus, the minimum ramp design speed should meet or exceed the design speed of the highway facility for which the through movement is provided. The design speed along the ramp will vary depending on alignment and controls at each end of the ramp. An acceptable approach is to set design speeds of 40 km/h and 80 km/h at the ramp terminus and exit nose, respectively, the appropriate design speed for any intermediate point on the ramp is then based on its location relative to those two points. When short radius curves with relatively lower design speeds are used, the vertical sight distance should be consistent with approach vehicle speeds. See Index 504.2(4) for additional information regarding design speed for ramps.

(b) Lane Width--**Ramp lanes shall be a minimum of 3.6 m in width. Where ramps have curve radii of 90 m or less with a central angle greater than 60 degrees, the single ramp lane, or the lane furthest to the right if the ramp is multilane, shall be widened in accordance with Table 504.3 in order to accommodate large truck**

wheel paths (see Topic 404). Consideration may be given to widening more than one lane on a multilane ramp with short radius curves if there is a likelihood of considerable bus or truck usage of that lane.

Table 504.3
Ramp Widening for Trucks

Ramp Radius (m)	Widening (m)	Lane Width (m)
<40	2.0	5.6
40 - 44	1.6	5.2
45 - 54	1.3	4.9
55 - 64	0.9	4.5
65 - 74	0.6	4.2
75 - 90	0.3	3.9
>90	0	3.6

(c) Shoulder Width--**Shoulder widths for ramps shall be as indicated in Table 302.1.** Typical ramp shoulder widths are 1.2 m on the left and 2.4 m on the right. The narrower widths indicated in Table 302.1 may be considered for multilane situations where blockage of one lane by a disabled vehicle will not create excessive queuing on the ramp.

(d) Lane Drops--Typically, lane drops are to be accomplished over a distance equal to 2/3WV. Where ramps are metered, the recommended lane drop taper past the meter limit line is 50 to 1. <u>Where conditions preclude the use of a 50 to 1 taper, the lane should be dropped using a taper of no less than 30 to 1</u>. **However, the lane drop taper past the limit line shall not be less than 15 to 1.**

<u>Lane drop tapers should not extend beyond the 2-meter point (the beginning of the weaving length) without the provision of an auxiliary lane.</u>

(e) Lane Additions -- Lane additions to ramps are usually accomplished by use of a 36 m bay taper. See Table 405.2A for the geometrics of bay tapers.

(f) Ramp Metering--The standards described in the HDM are to be applied to both metered and non-metered ramps. However, attention is directed to the "Ramp Meter Design Guidelines". These guidelines, when used in conjunction with HDM standards, should provide adequate guidance in the design of metered ramps. Ramp Meter Development Plans, developed by the District Traffic Operations unit should be consulted to determine if entrance ramps need to be designed to accommodate future metering hardware and/or future lane configurations.

(2) Location and Design of Ramp Intersections on the Crossroads.

Factors which influence the location of ramp intersections on the crossroads include sight distance, construction and right of way costs, circuitry of travel for left-turn movements, crossroads gradient at ramp intersections, storage requirements for left-turn movements off the crossroads, and the proximity of other local road intersections.

<u>Ramp terminals should connect where the grade of the overcrossing is 4% or less to avoid potential overturning of trucks.</u>

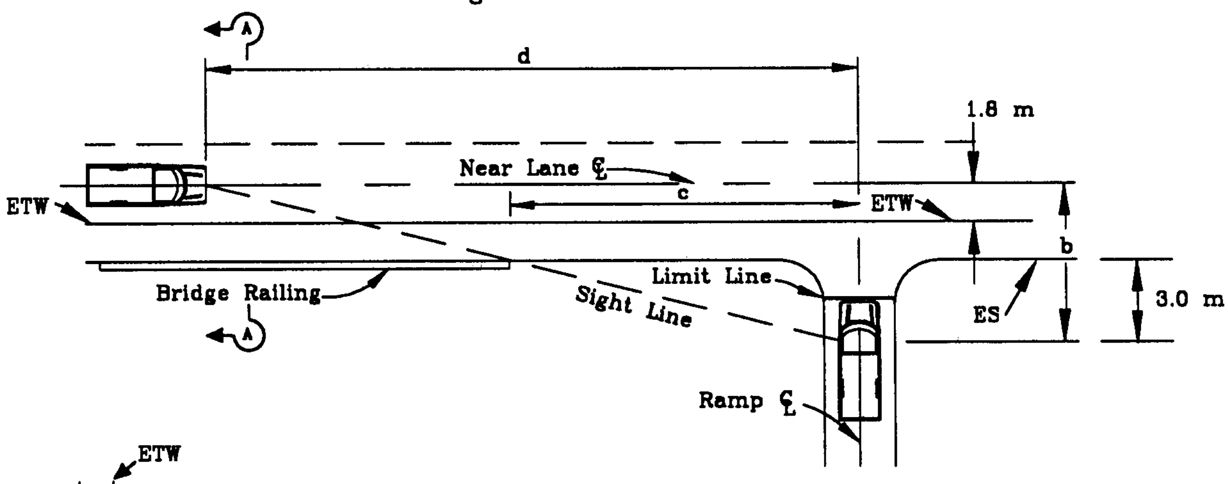
<u>For left-turn maneuvers from an off-ramp at an unsignalized intersection, the length of crossroads open to view should be greater than the product of the prevailing speed of vehicles on the crossroads, and the time required for a stopped vehicle on the ramp to execute a left-turn maneuver. This time is estimated to be 7-1/2 seconds.</u>

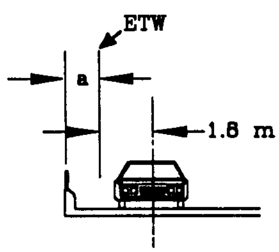
<u>Where a separate right turn lane is provided at ramp terminals, the turn lane should not continue as a "free" right unless pedestrian volumes are low, the right turn lane continues as a separate full width lane for at least 60 m prior to merging and access control is maintained for at least 60 m past the ramp intersection. Provision of the "free" right should also be precluded if left turn movements of any kind are allowed within 125 m of the ramp intersection.</u>

Figure 504.3A

Location of Ramp Intersections on the Crossroads

Unsignalized and based on 7.5 second
horizontal sight distance criteria





SECTION A - A

$$c=d\left[\frac{b-(a+1.8\ \text{m})}{b}\right]$$

V (km/h)	d (m)
40	90
50	110
60	130
70	150
80	170
90	190
100	210
110	230

a=Distance from edge of traveled way to bridge railing.

b=Distance from center of near lane to eye of ramp vehicle driver. Ramp driver's eye is assumed to be located 3 m from the edge of shoulder, but not less than 4 m from the ETW. (Therefore, b = 1.8 m + shoulder width + 3.0 m) See Index 405.1.

c=Ramp set back from end of bridge railing.

d=Corner Sight distance along highway from intersection. (See Table above.) Sight distance is measured from a 1070 mm eye height on the ramp to a 1300 mm object height on the crossroad.

V=Anticipated prevailing speed on crossroad.

Horizontal sight restrictions may be caused by bridge railings, bridge piers, or slopes. Sight distance is measured between the center of the outside lane approaching the ramp and the eye of the driver of the ramp vehicle assumed 3.0 m back from the edge of shoulder at the crossroads. Figure 504.3A illustrates the determination of ramp setback from an overcrossing structure on the basis of sight distance controlled by the bridge rail. The same relationship exists for sight distance controlled by bridge piers or slopes.

Where ramp set back for the 7-1/2 second criteria is unobtainable, sight distance should be provided by flaring the end of the overcrossing structures or setting back the piers or end slopes of an undercrossing structure.

If signals are warranted within 5 years of construction, consideration may be given to installing signals initially in lieu of providing the 7-1/2 second horizontal sight distance. However, this is not desirable and corner sight distance commensurate with design speed should be provided where obtainable (see AASHTO, A Policy on Geometric Design of Highways and Streets, 1994).

For additional information on sight distance requirements at signalized intersections, see Index 405.1.

For new construction or major reconstruction of interchanges, the minimum distance between ramp intersections and local road intersections shall be 125 m. <u>The preferred minimum distance should be 160 m.</u> This does not apply to Resurfacing, Restoration and Rehabilitation (RRR), ramp widening, restriping or other projects which do not reconfigure the interchange. This standard does apply to projects proposing to realign a local street.

Where intersections are closely spaced, traffic operations are often inhibited by short weave and storage lengths, and signal phasing. In addition it is difficult to provide proper signing and delineation. Whenever it becomes necessary to locate a ramp terminal close to an intersection, the District Traffic Branch should be consulted regarding the requirement for signing, delineation and signal phasing.

(3) *Superelevation for Ramps*. The factors controlling superelevation rates discussed in Topic 202 apply also to ramps. As indicated in Table 202.2 use the 12% e_{max} rate except where snow and ice conditions prevail. In restrictive cases where the length of curve is too short to develop standard superelevation, the highest obtainable rate should be used (see Index 202.5). If feasible, the curve radius can be increased to reduce the standard superelevation rate. Both edge of traveled way and edge of shoulder should be examined at ramp junctions to assure a smooth transition.

Under certain restrictive conditions the standard superelevation rate discussed above may not be required on the last curve (near the ramp intersection) of a multi-curve diagonal ramp. The specific conditions under which lower superelevation rates would be considered must be evaluated on a case-by-case basis and must be discussed with the Project Development Coordinator. Documentation shall be as required by the Coordinator.

(4) *Single-lane Ramps.* Single lane ramps are those ramps that either enter into or exit from the freeway as a single lane. These ramps are often widened near the ramp intersection with the crossroads to accommodate turning movements onto or from the ramp. <u>When additional lanes are provided near the entrance ramp intersection the lane drop should be accomplished over a distance equal to 2/3WV. The lane to be dropped should be on the right so that traffic merges left.</u>

Exit ramps in metropolitan areas may require multiple lanes at the intersection with the crossroads to provide additional storage and capacity. <u>If the length of a single lane ramp exceeds 300 m, an additional lane should be provided on the ramp to permit passing maneuvers.</u> Figure 504.3B illustrates alternative ways of transitioning a single lane exit ramp to two lanes. The decision to use Alternate A or Alternate B is generally based on providing the additional lane for the minor movement.

Figure 504.3B
Transition From a Single-Lane Exit Ramp to Two Lanes

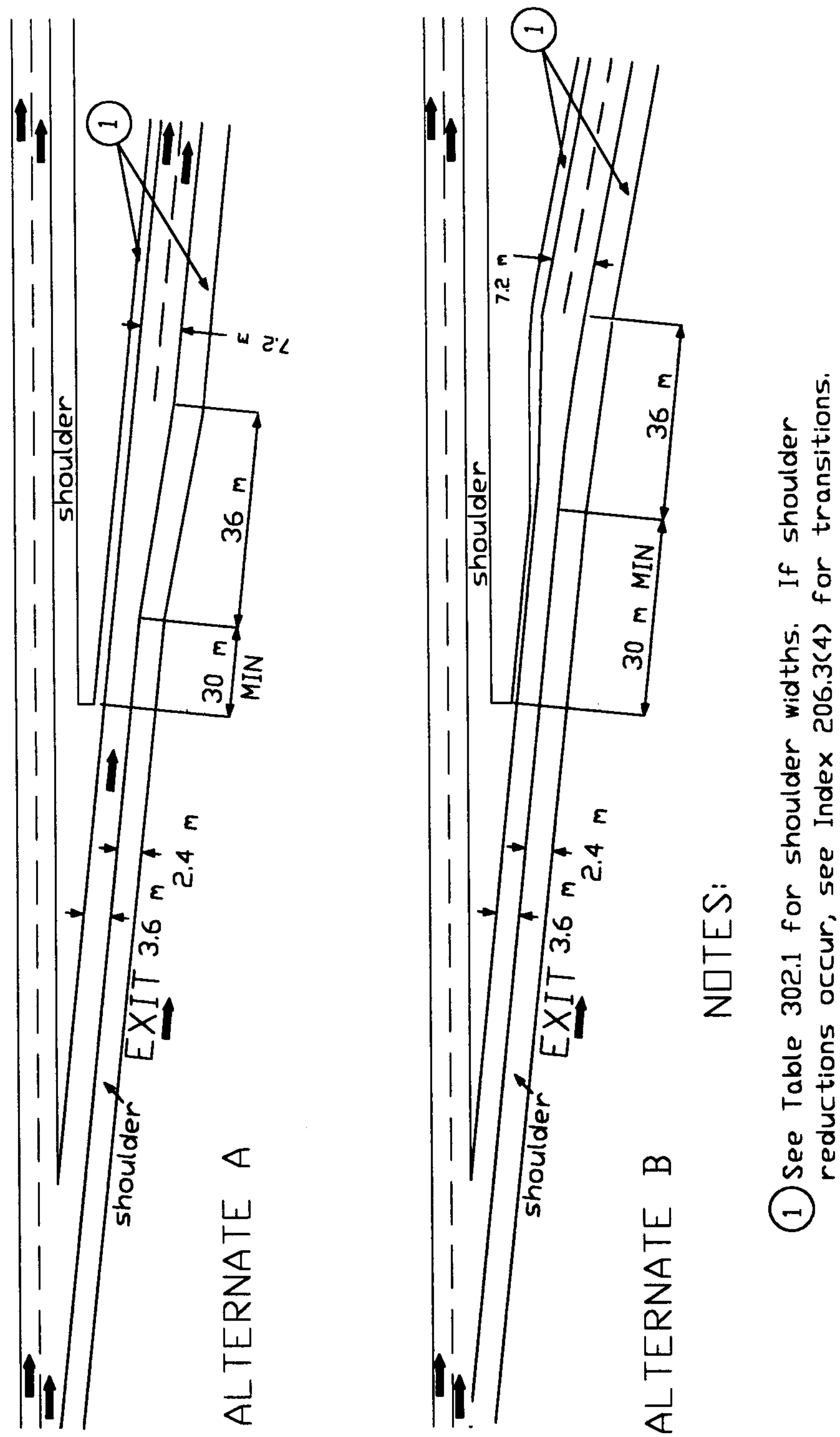
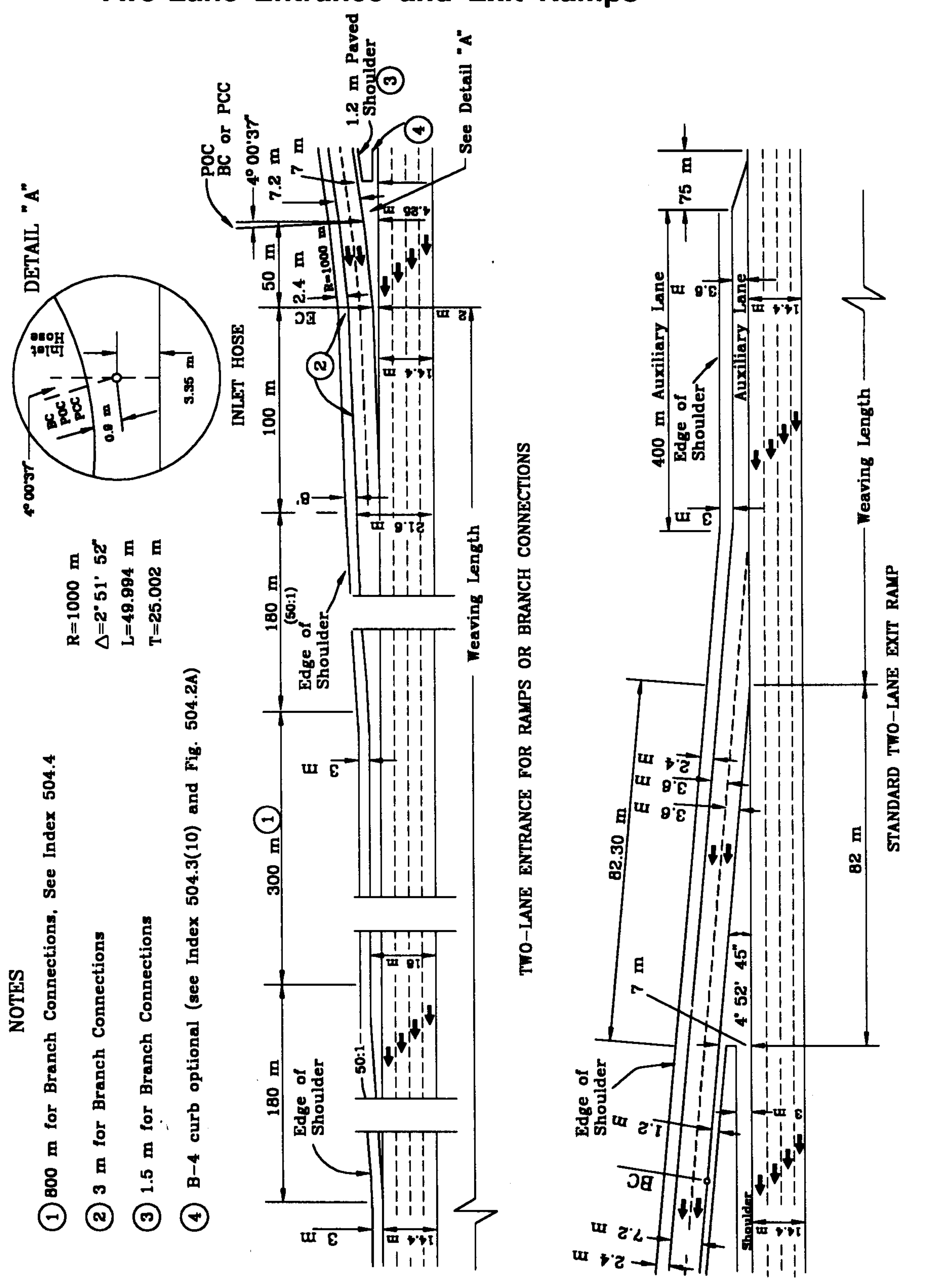


Figure 504.3C

Two-Lane Entrance and Exit Ramps



(5) *Two-lane Exit Ramps*. Where design year estimated volumes exceed 1500 equivalent passenger cars per hour, a 2-lane width of ramp should be provided initially.

Provisions should be made for possible widening to three or more lanes at the crossroads intersection. Figure 504.3C illustrates the standard design for a 2-lane exit. An auxiliary lane approximately 400 m long should be provided in advance of a 2-lane exit. For volumes less than 1500 but more than 900, a one-lane width exit ramp should be provided with provision for adding an auxiliary lane and an additional lane on the ramp.

(6) *Two-lane Entrance Ramps*. A standard two lane entrance ramp is illustrated in Figure 504.3C. This design may be utilized in situations where the estimated design year volume exceeds 1500 equivalent passenger cars per hour. The configuration shown in Figure 504.3C, which includes the provision of a 300 m auxiliary lane parallel to the freeway, will typically only be used where adequate capacity exists on the through facility in the design year. Where capacity is limited, consideration should be given to extending the auxiliary lane to the next interchange or adding additional lanes to the freeway. For most urban situations, it is recommended that multiple ramp lanes taper to a single lane prior to the 2-meter separation point (where merging is considered to begin). A thorough investigation of ramp volumes versus through facility volumes must be made for off-peak as well as peak periods if metering of the ramp is anticipated. Early discussion with the Headquarters Traffic Reviewer and Project Development Coordinator or Geometric Reviewer is recommended whenever two lane entrance ramps are being considered.

(7) *Loop Ramps*. Normally, loop ramps should have one lane and shoulders unless a second lane is needed for capacity or ramp metering purposes. Consideration should be given to providing a directional ramp when loop volumes exceed 1500 vehicles per hour. If two lanes are provided, normally only the right lane needs to be widened for trucks. See Topic 404 for additional discussion on lane widths and design of ramp intersections to accommodate the design vehicle. See Index 504.3(1) for a discussion on ramp widening for trucks.

Radii for loop ramps should normally range from 45 m to 60 m. Increasing the radii beyond 60 m is typically not cost effective as the slight increase in design speed is usually outweighed by the increased right of way requirements and the increased travel distance. Curve radii of less than 35 m should also be avoided. Extremely tight curves lead to increased off-tracking by trucks and increase the potential for vehicles to enter the curve with excessive speed.

Of particular concern in the design of loop ramps are the constraints imposed on large trucks. Research indicates that trucks often enter loops with excessive speed, either due to inadequate deceleration on exit ramps or due to driver efforts to maintain speed on entrance ramps to facilitate acceleration and merging. Where the loop is of short radius and is also on a steep descent (over 6%), it is important to develop the standard 2/3 full superelevation rate by the beginning of the curve (see Index 504.2(5)). On loop entrance ramps this can often be facilitated by beginning the ramp with a short tangent (20 m to 30 m) that diverges from the cross street at an angle of 4 to 9 degrees. Consideration should be given to developing additional tangent length if conditions allow.

The ramp lane structural section should be provided on shoulders for curves with a radius less than 90 m (see Index 608.6).

(8) *Distance Between Successive On-ramps.* The minimum distance between two successive on-ramps to a freeway lane should be the distance needed to provide the standard on-ramp acceleration taper shown on Figure 504.2A. This distance should be about 300 m unless the upstream ramp adds an auxiliary lane in which case the downstream ramp should merge with the auxiliary lane in a standard 50:1 convergence. The distance between on-ramp noses will then be controlled by interchange geometry.

(9) *Distance Between Successive Exits*. The minimum distance between successive exit ramps for guide signing should be 300 m on

the freeway and 180 m on collector-distributor roads.

(10) *Curbs.* Curbs should not be used on ramps except in the following locations:

(a) An 80 mm concrete curb (Type B-4, Index 209.2), may be used on both sides of the separation between freeway lanes and a parallel collector-distributor road.

(b) A B-4 curb may be used as shown in Figure 504.2A to control drainage or where the gore cross slope would be greater than allowed in Index 504.3(3). When the optional B-4 curb is used at the entrance ramp inlet nose, the shoulder adjacent to the curb should be the same width as the ramp shoulder approaching the curb. The B-4 gutter pan can be included as part of the shoulder width. As stated in Index 405.4(2), curbs are typically discouraged where design speeds are over 75 km/h. The appropriateness of curbs at gore areas must be determined on a case-by-case basis.

(c) Curbs may be used where necessary at the ramp connection with the local street for the protection of pedestrians, for channelization, and to provide compatibility with the local facility.

(d) Curbs may be used where necessary to control drainage.

(e) The Type E curb may be used only in special drainage situations, for example, where drainage parallels and flows against the face of a retaining wall. In general, curbs should not be used on the high side of ramps or in off-ramp gore areas except at collector-distributor roads.

(11) *Dikes.* The use of dikes should conform to the requirements of Index 835.3.

(12) *Ramp Capacity.* See Chapter 5 of the Highway Capacity Manual for an analysis of ramp capacity.

504.4 Freeway-to-Freeway Connections

(1) *General.* All of the design criteria discussed in Indexes 501.3, 504.2 and 504.3 apply to freeway to freeway connectors, except as discussed or modified below.

(2) *Design Speed.* The design speed for single lane directional and all branch connections should be a minimum of 80 km/h. When smaller radius curves, with lower design speeds, are used the vertical sight distance should be consistent with approaching vehicle speeds. Design speed for loop connectors should be consistent with Index 504.2(4).

(3) *Grades.* The maximum profile grade on freeway-to-freeway connections should not exceed 6%. Flatter grades and longer vertical curves than those used on ramps are needed to obtain increased stopping sight distance for higher design speeds.

(4) *Shoulder Width.*

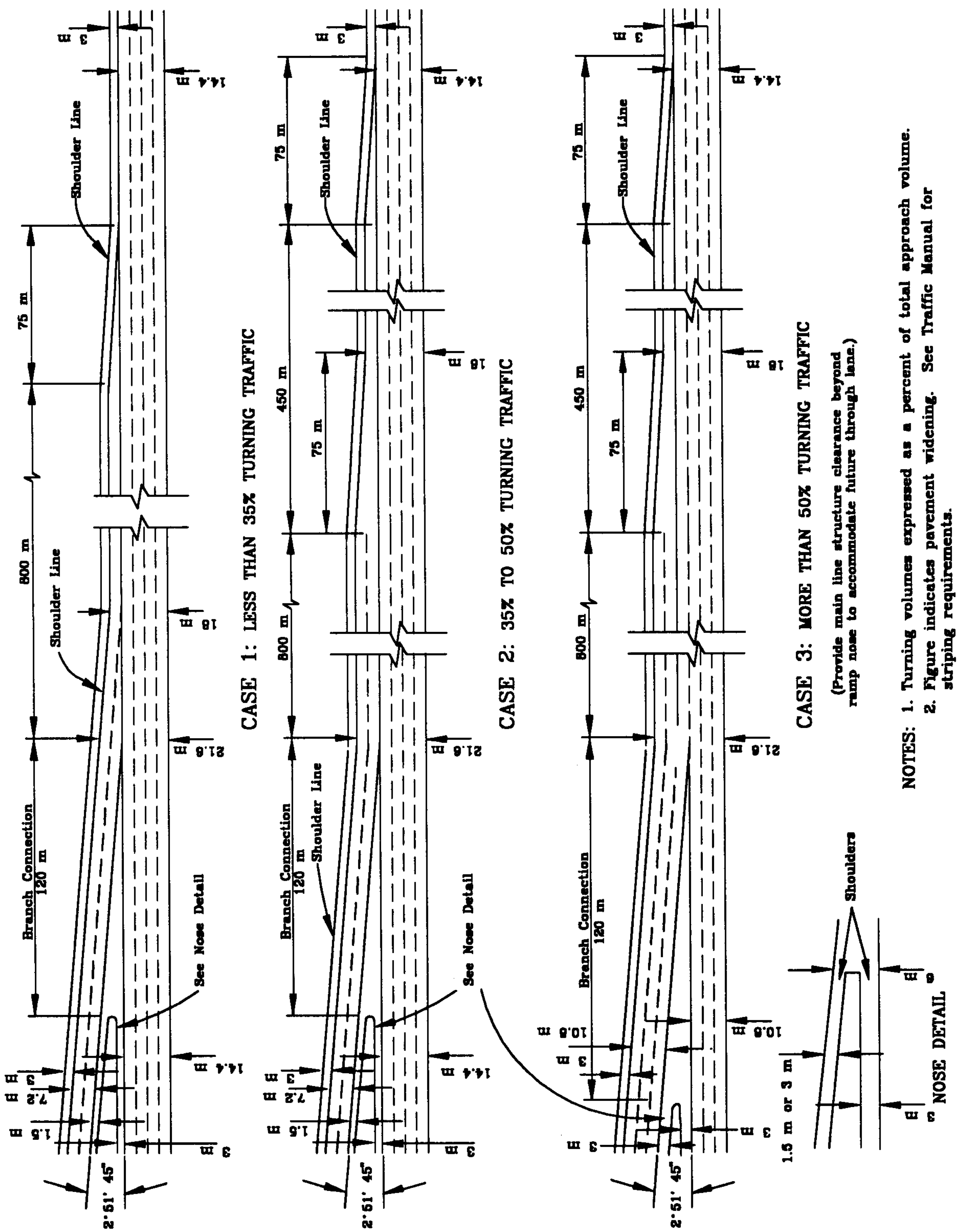
(a) Single-lane and Two-lane Connections--**The width of shoulders on single-lane and two-lane (except as described below) freeway-to-freeway connectors shall be 1.5 m on the left and 3.0 m on the right. A single lane freeway-to-freeway connector that has been widened to two lanes solely to provide passing opportunities and not due to capacity requirements shall have a 1.5 m left shoulder and at least a 1.5 m right shoulder** (see Index 504.4(5)).

(b) Three-lane Connections--**The width of shoulders on three-lane connectors shall be 3.0 m on both the left and right sides.**

(5) *Single-lane Connections.* Freeway-to-freeway connectors may be single lane or multilane. Where design year volume is between 900 and 1500 equivalent passenger cars per hour, initial construction should provide a single lane connection with the capability of adding an additional lane. Single lane directional connectors should be designed using the general configurations shown on Figure 504.2A, but utilizing the flatter diverge angle shown in Figure 504.4. Single lane loop connectors may use a diverge angle of as much as that shown on Figure 504.2A for ramps, if necessary. The

Figure 504.4

Diverging Branch Connections



NOTES: 1. Turning volumes expressed as a percent of total approach volume.
2. Figure indicates pavement widening. See Traffic Manual for striping requirements.

choice will depend upon interchange configuration and driver expectancy. Single lane connectors in excess of 300 m in length should be widened to two lanes to provide for passing maneuvers (see Index 504.4(4)).

(6) *Branch Connections.* A branch connection is defined as a multilane connection between two freeways. A branch connection should be provided when the design year volume exceeds 1500 equivalent passenger cars per hour.

Merging branch connections should be designed as shown in Figure 504.3C. Diverging branch connections should be designed as shown in Figure 504.4. The diverging branch connection leaves the main freeway lanes on a flatter angle than the standard 2-lane ramp exit connection shown in Figure 504.3B. The standard ramp exit connects to a local street. The diverging branch connection connects to another freeway and has a flatter angle that allows a higher departure speed.

At a branch merge, an 800 m length of auxiliary lane should be provided beyond the merge of one lane of the inlet, except where it does not appear that capacity on the freeway will be reached until five or more years after the 20 year design period. In this case the length of auxiliary lane should be a minimum of 300 m. For diverging connections where less than capacity conditions beyond the design year are anticipated, the length of auxiliary lane in advance of the exit should be 400 m.

(7) *Lane Drops.* The lane drop taper on a freeway-to-freeway connector should not be less than 2/3WV.

(8) *Metering.* Any decision to meter freeway-to-freeway connectors must be carefully considered as driver expectancy on these types of facilities is for high-speed uninterrupted flow. If metering is anticipated on a connector, discussions with the Headquarters Traffic Reviewer and Project Development Coordinator should take place as early as possible. Issues of particular concern are adequate deceleration lengths to the end of the queue, potential need to widen shoulders if sight distance is restricted (particularly on ramps with 1.5 m shoulders on each side), and the potential for queuing back onto the departing freeway.

504.5 Auxiliary Lanes

In order to ensure satisfactory operating conditions, auxiliary lanes may be added to the basic width of traveled way.

Where an entrance ramp of one interchange is closely followed by an exit ramp of another interchange, the acceleration and deceleration lanes should be joined with an auxiliary lane. Auxiliary lanes should be provided in all cases when the weaving distance, measured as shown in Figure 504.2A, is less than 600 m. Where interchanges are more widely spaced and ramp volumes are high, the need for an auxiliary lane between the interchanges should be determined in accordance with Index 504.7.

Auxiliary lanes may be used for the orientation of traffic at 2-lane ramps or branch connections as illustrated on Figure 504.3C and Figure 504.4. The length and number of auxiliary lanes in advance of 2-lane exits are based on percentages of turning traffic and a weaving analysis.

Auxiliary lanes may be warranted when merging an ascending entrance ramp with high truck volumes onto a mainline facility with a sustained upgrade. An auxiliary lane would allow entrance ramp traffic to accelerate to a higher speed before merging with mainline traffic, or simply provide more opportunity to merge. See Index 504.2 for specific requirements.

504.6 Mainline Lane Reduction at Interchanges

The basic number of mainline lanes should not be dropped through a local service interchange. The same standard should also be applied to freeway-to-freeway interchanges where less than 35% of the traffic is turning (see Figure 504.4). Where more than 35% of the freeway traffic is turning, consideration may be given to reducing the number of lanes. No decision to reduce the number of lanes should be made without the approval of the District Traffic Operations Unit. Additionally, adequate structure clearance (both horizontal and vertical) should be provided to accommodate future construction of the dropped lane if required.

Where the reduction in traffic volumes is sufficient to warrant a decrease in the basic number of lanes, a preferred location for the lane drop is beyond the influence of an interchange and preferably at least 1 km from the nearest exit or inlet nose. It is desirable to drop the right lane on tangent alignment with a straight or sag profile so vehicles can merge left with good visibility to the pavement markings in the merge area (see Index 201.7).

504.7 Weaving Sections

A weaving section is a length of one-way roadway where vehicles are crossing paths, changing lanes, or merging with through traffic as they enter or exit a freeway or collector-distributor road.

A single weaving section has an inlet at the upstream end and an exit at the downstream end. A multiple weaving section is characterized by more than one point of entry followed by one or more points of exit.

A rough approximation for adequate length of a weaving section is 0.3 m of length per weaving vehicle per hour. This rate will approximately provide a level of service C. Refer to the January 31, 1995 Design Information Bulletin Number 77 on Interchange Spacing for additional weaving requirements.

There are various methods for analyzing weaving sections. Three methods which provide valid results are described below.

The Leisch method, which is usually considered the easiest to use, is illustrated in Figure 504.7A. This method was developed by Jack Leisch & Associates and may be used to determine the length of weaving sections for both freeways and collector-distributor roads. The Leisch weaving charts determine the level of service for the weaving volumes for the length of the weaving section from the first panel on the lower left of the chart. The analysis is dependent on whether the section is balanced or unbalanced, as defined in Figure 504.7B. The level of service for the total volume over all lanes of the weaving section is then found from the panels on the right of the chart. The weaving chart should not be extrapolated.

Pages 234-238 of the 1965 Highway Capacity Manual (HCM) provide a method for determining the adequacy of weaving sections near single lane ramps. It is often referred to as the Level of Service (LOS) D method. This method is also documented in Traffic Bulletin 4 which is available from the Traffic Operations Program. The LOS D method can be used to project volumes along a weaving section. These volumes can be compared to the capacities along the same weaving section. Volumes in passenger car equivalents per hour (PCEPH) should be adjusted for freeway grade and truck volumes. Table 504.7C and Figures 504.7D and E are reprinted from the 1965 HCM and provide information regarding vehicle distribution by lane.

The results obtained form Figure 504.7A (the Leisch Method) for single-lane ramps with an auxiliary lane and weaving rates exceeding 2500 PCEPH should be checked using the LOS D method.

A method for analyzing multilane weaving sections has been developed by the Institute of Transportation Studies, University of California, Berkeley. This methodology is similar to the Level of Service D approach and PC compatible software is available to perform the analysis. The District Traffic Operations unit should have copies of the software and have information on the applicability of the program.

Weaving capacity analyses other than those described above should not be used on California highways. Other methods, such as the one contained in the 1994 HCM, may not always produce accurate results.

<u>Weaving sections in urban areas should be designed for level of service C or D. Weaving sections in rural areas should be designed for level of service B or C. Design rates for lane balanced weaving sections where at least one ramp or connector will be two lanes should not result in a level of service lower than the middle of level of service D using Figure 504.7A</u>. In determining acceptable hourly operating volumes, peak hour factors should be used.

<u>On main freeway lanes the weaving length measured as shown in Figure 504.2A should not be less than 500 m except where excessive cost or severe environmental constraints would require consideration of a shorter length. 300 m of length should be added for each additional</u>

lane to be crossed by weaving vehicles. It should be noted that a weaving analysis must be considered over an entire freeway segment as weaving can be affected by other nearby ramps.

The District Traffic Operations Branch should be consulted for difficult weaving analysis problems.

504.8 Access Control

Access rights shall be acquired along interchange ramps to their junction with the nearest public road. At such junctions, for new construction, access control should extend 30 m beyond the end of the curb return or ramp radius in urban areas and 100 m in rural areas, or as far as necessary to ensure that entry onto the facility does not impair operational characteristics. **Access control shall extend at least 15 m beyond the end of the curb return, ramp radius, or taper.**

Typical examples of access control at interchanges are shown in Figure 504.8. These illustrations do not presume to cover all situations or to indicate the most desirable designs for all cases. Whenever there is access control on both sides of a local street, the State owns that R/W and a maintenance agreement is needed.

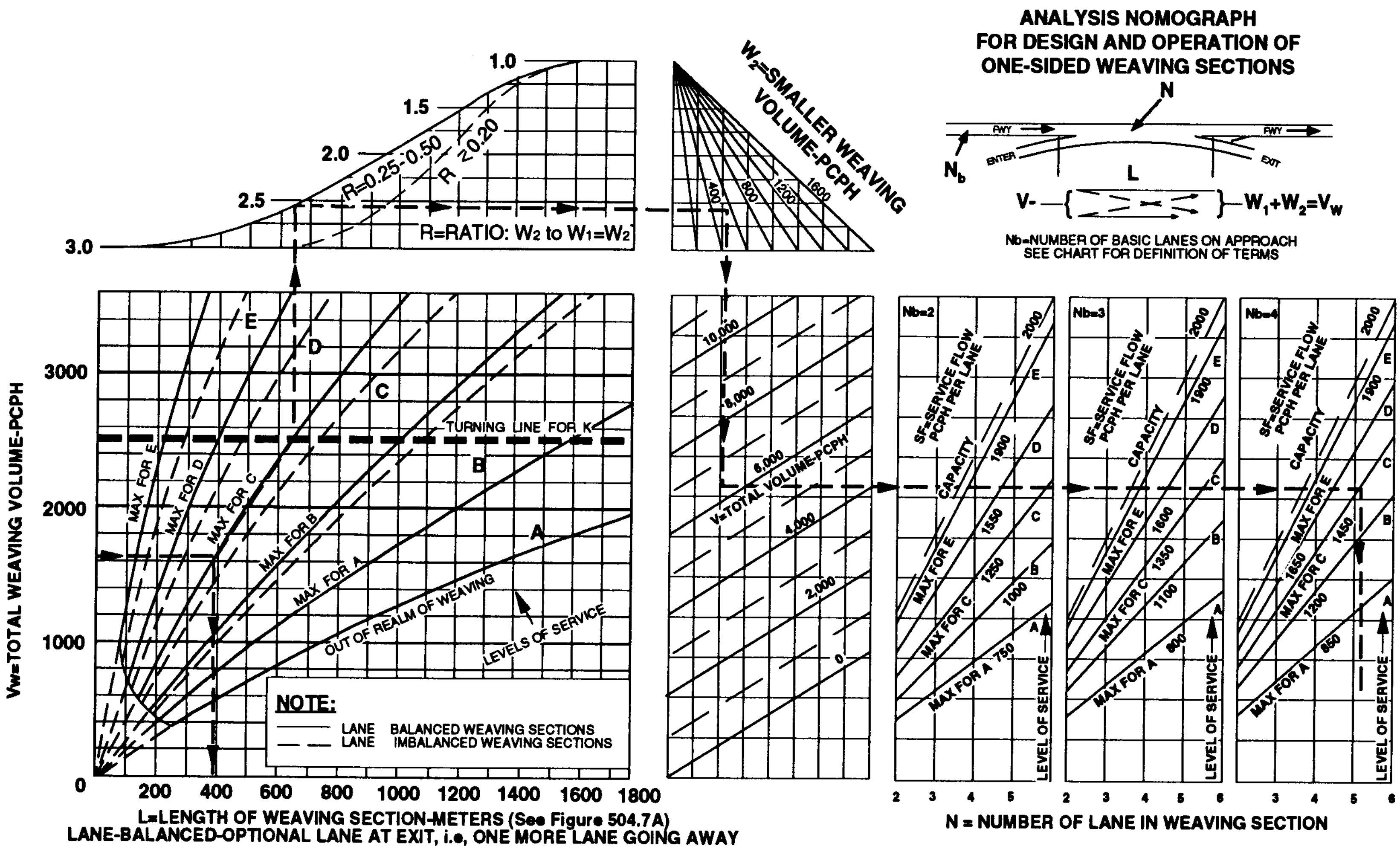
It is also desirable to obtain access control on the opposite side of the local road from ramp terminals to preclude the construction of future driveways or local roads within the ramp intersection. This access control would limit the volume of traffic and the number of phases at the intersection of the ramp and local facility, thereby optimizing capacity and operation of the ramp. Through a combination of access control and the use of raised median islands along the local facility, intersections should be located at least 125 m from the ramp intersection. Right in - right out access may be permitted beyond 60 m from the ramp intersection. The length of access control on both sides of the local facility should match.

In Case 2 consider private ownership within the loop only if access to the property is an adequate distance from the ramp junction to preserve operational integrity.

In Case 3 if the crossroads is near the ramp junction at the local road it is recommended that full access control be acquired on the local street from the junction to the intersection with the crossroads.

Case 6 represents a slip ramp design. If the ramp is perpendicular to the local/frontage road refer to Case 3. In Case 6 if the crossroads is near the ramp junction to the local road it is recommended that access control be acquired on the opposite side of the local street from the junction.

Figure 504.7A

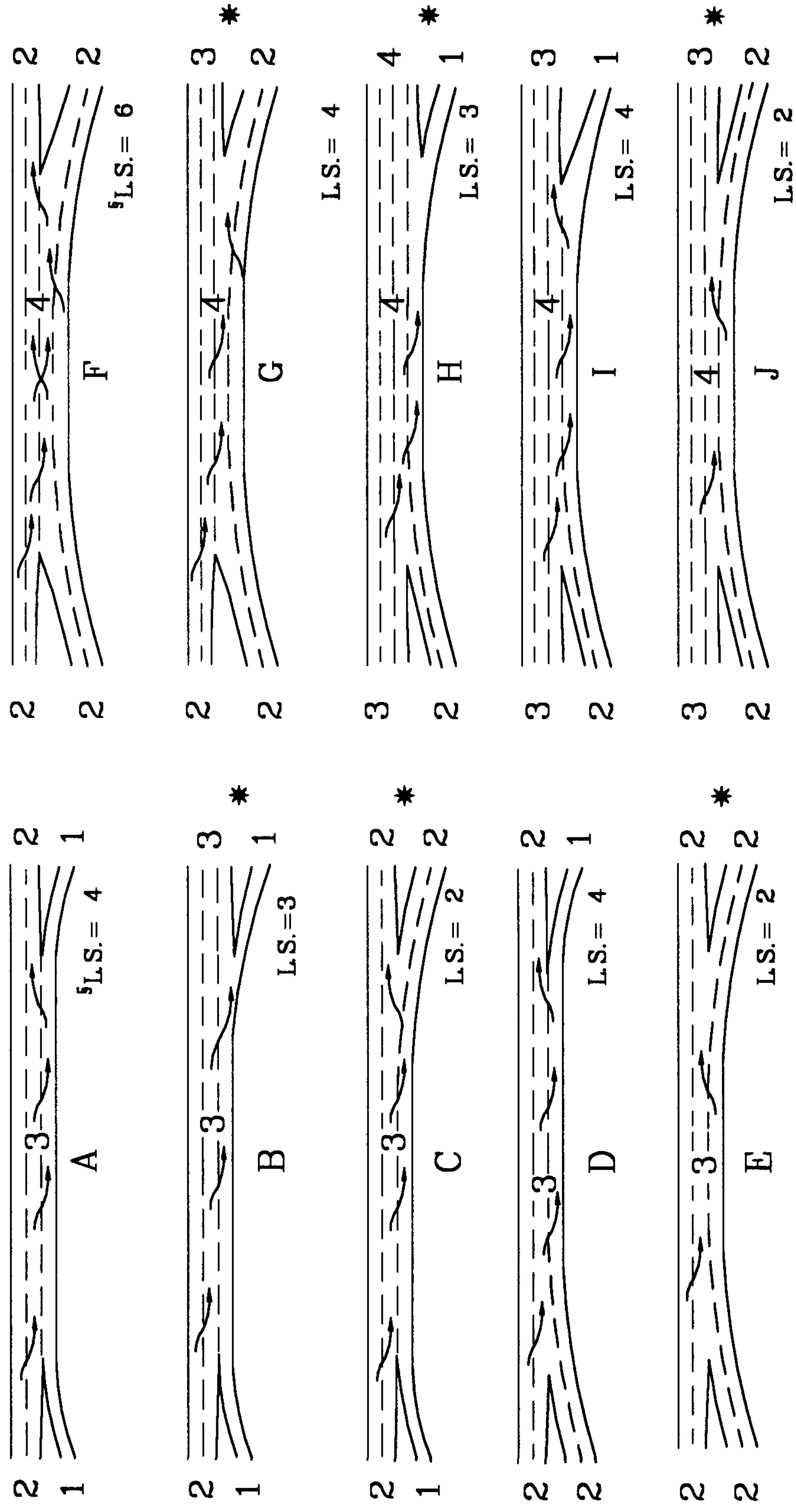


Source: Jack E. Leisch & Associates

NOTE: EXTRAPOLATION OF CHART BEYOND THE BOUNDARIES GIVEN IS NOT ADVISED.

Example: The nomograph is entered on the left (see dashed line and arrows) with weaving volume, W_1+W_2 (or V_W) followed by projection to the right, intersecting the desired weaving LOS: a vertical drop from this point provides weaving distance L = 400 m. Returning to first intersetion point of V_W with LOS line, an upward projection along the LOS is intersected with the horizontal, heavy dashed, "turning line for K:" from here the solution line is extended vertically to intersect the K values curve, from which a horizontal extension meets the desired W_2 volume. Then a downward turn to total volume, V, from which the line is horizontally projected to the right, intersection (in this case) the desired LOS = C curve having an SF of 1450 (representing the overall or composite operation of the weaving section), from which a downward extension yields a N of 5.2: this would be rounded to N = 5 lanes.

Figure 504.7B
Lane Configuration of Weaving Sections



Source: Jack E. Leisch & Associates

* DENOTE LANE BALANCE – OPTIONAL LANE AT EXIT

^{5}L.S. – POTENTIAL LANE SHIFTS, CONSIDERING MAX OF 2 LANES INVOLVED ON ANY ONE APPROACH

Table 504.7C

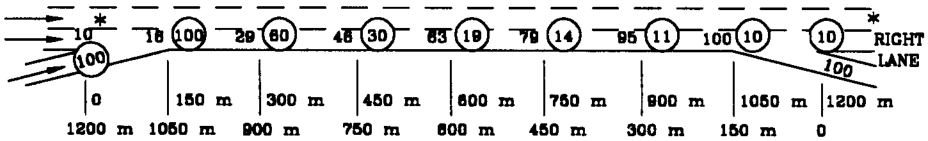
Percent of Through Traffic Remaining in Outer Through Lane
(Level of Service D Procedure)

TOTAL VOLUME OF THROUGH TRAFFIC, ONE DIRECTION (vph)	APPROXIMATE PERCENTAGE OF THROUGH[a] TRAFFIC REMAINING IN THE OUTER THROUGH LANE IN THE VICINITY OF RAMP TERMINALS AT LEVEL OF SERVICE D.		
	8-LANE[b] FREEWAY	6-LANE[c] FREEWAY	4-LANE[d] FREEWAY
6500 and over	10	-	-
6000 - 6499	10	-	-
5500 - 5999	10	-	-
5000 - 5499	9	-	-
4500 - 4999	9	18	-
4000 - 4499	8	14	-
3500 - 3999	8	10	-
3000 - 3499	8	6	40
2500 - 2999	8	6	35
2000 - 2499	8	6	30
1500 - 1999	8	6	25
Up to 1499	8	6	20

a. Traffic not involved in a ramp movement within 1200 m in either direction.
b. 4 lanes one way
c. 3 lanes one way
d. 2 lanes one way

Figure 504.7D
Percentage Distribution of On- and Off-ramp Traffic in Outer Through Lane and Auxiliary Lane
(Level of Service D Procedure)

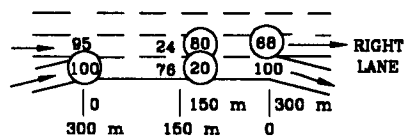
CASE I - SINGLE - LANE ON- AND OFF-RAMPS WITHOUT AUXILIARY LANE
(THIS CHART MAY BE USED REGARDLESS OF ACTUAL SPACING BETWEEN ON- AND OFF-RAMPS, BUT AS NOTED BELOW* CAUTION MUST BE EXCERCISED IN USING THESE VALUES.



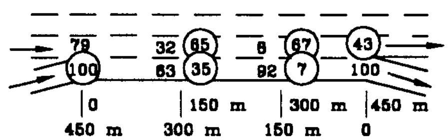
CASE II - SINGLE - LANE - ON- AND OFF-RAMPS WITH AUXILIARY LANE**

(A) L = 300 m

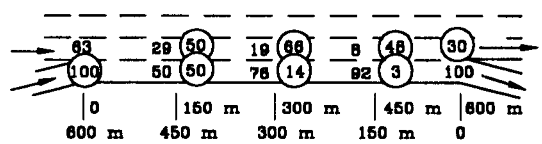
> **EXAMPLE:**
> GIVEN: L = 300 m
> PORTION OF V_1 THROUGH
> (FROM TABLE 504.7C = 475 VPH
> ON-RAMP = 1,000 VPH
> OFF-RAMP = 1,200 VPH
> ON-RAMP TO OFF-RAMP = 0
> FIND: V_1 (VOL. IN OUTHER THROUGH LANE) @ 150 m =
> 475 + (0.60)(1,000) + (0.24)(1,200) =
> 1,563 VPH



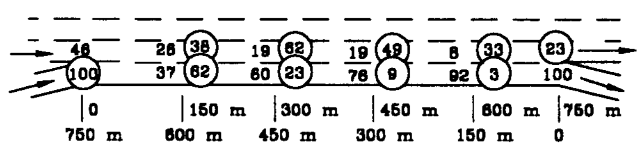
(B) L = 450 m



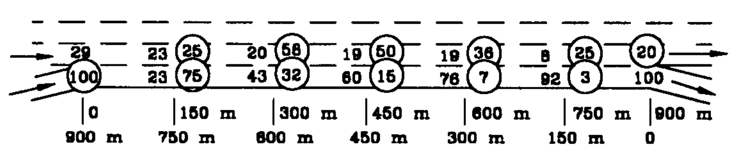
(C) L = 600 m



(D) L = 750 m



(E) L = 900 m



CIRCLED VALUES (0) INDICATE PERCENTAGE OF ON-RAMP TRAFFIC IN LANE SHOWN. UNCIRCLED VALUES INDICATE PERCENTAGE OF OFF-RAMP TRAFFIC IN LANE SHOWN. (REMAINING PORTION OF TRAFFIC IS IN LANE(S) TO LEFT OF OUTER THROUGH LANE.)

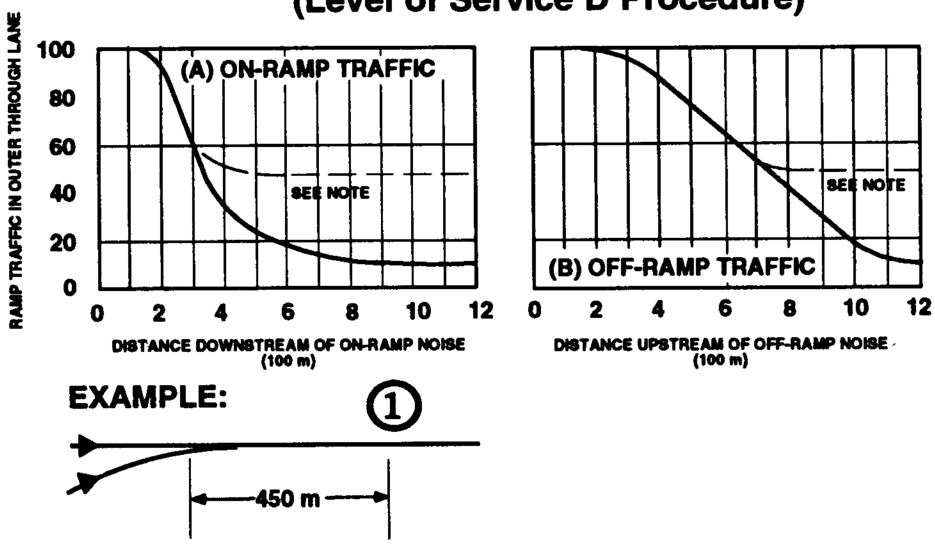
THESE PERCENTAGES ARE NOT NECESSARILY THE DISTRIBUTIONS UNDER FREE FLOW OR LIGHT RAMP TRAFFIC, BUT UNDER PRESSURE OF HIGH VOLUMES IN THE RIGHT LANES AT THE POINT BEING CONSIDERED AND WITH ROOM AVAILABLE IN OTHER LANES.

* MINIMUM % IN RIGHT LANE CANNOT BE LESS THAN % OF THROUGH TRAFFIC IN RIGHT LANE AS DETERMINED FROM TABLE 504.7C (SEE NOTE, FIG. 5047E).

** SEE FIGURE 504.2A FOR METHOD OF MEASURING LENGTH L (WEAVING LENGTH).

Figure 504.7E

Percentage of Ramp Traffic in the Outer Through Lane (No Auxiliary Lane) (Level of Service D Procedure)



A - NORMAL CALCULATION
2 LANES ONE-WAY
"THROUGH TRAFFIC" = 2,400 VPH
"ON-RAMP" = 800 VPH

AMOUNT IN THE OUTER THROUGH LANE AT ①
THROUGH (FROM TABLE 504.7C) = 0.30 X 2,400 = 720
ON-RAMP (FROM CHART ABOVE) = 0.30 X 800 = 240
960

B - CHECK CALCULATIONS
BECAUSE % IN THE OUTER THROUGH LANE AT 450 M IS BELOW DASHED LINE, RECALCULATE ASSUMING ON-RAMP TRAFFIC IS THROUGH TRAFFIC.

AMOUNT IN THE OUTER THROUGH LANE AT ①
THROUGH (FROM TABLE 504.7C) 0.40 X 3,200 = 1,280
SINCE CALCULATION B (1,280) IS GREATER THAN CALCULATION A (960) USE 1,280.

*THESE PERCENTAGES ARE NOT NECESSARILY THE DISTRIBUTIONS UNDER FREE FLOW OR LIGHT RAMP TRAFFIC, BUT UNDER PRESSURE OF HIGH VOLUMES IN THE RIGHT LANES AT THE LOCATION BEING CONSIDERED AND WITH AVAILABLE ROOM IN OTHER LANES.

NOTE: IF RAMP PERCENTAGE IN THE OUTER THROUGH LANE AT POINT UNDER CONSIDERATION IS BELOW DASHED LINE, THEN AMOUNT IN THE OUTER THROUGH LANE SHOULD BE RECALCULATED ASSUMING RAMP TRAFFIC IS THROUGH TRAFFIC. USE HIGHER VALUE. SEE EXAMPLE ABOVE.

Figure 504.8

Typical Examples of Access Control at Interchanges

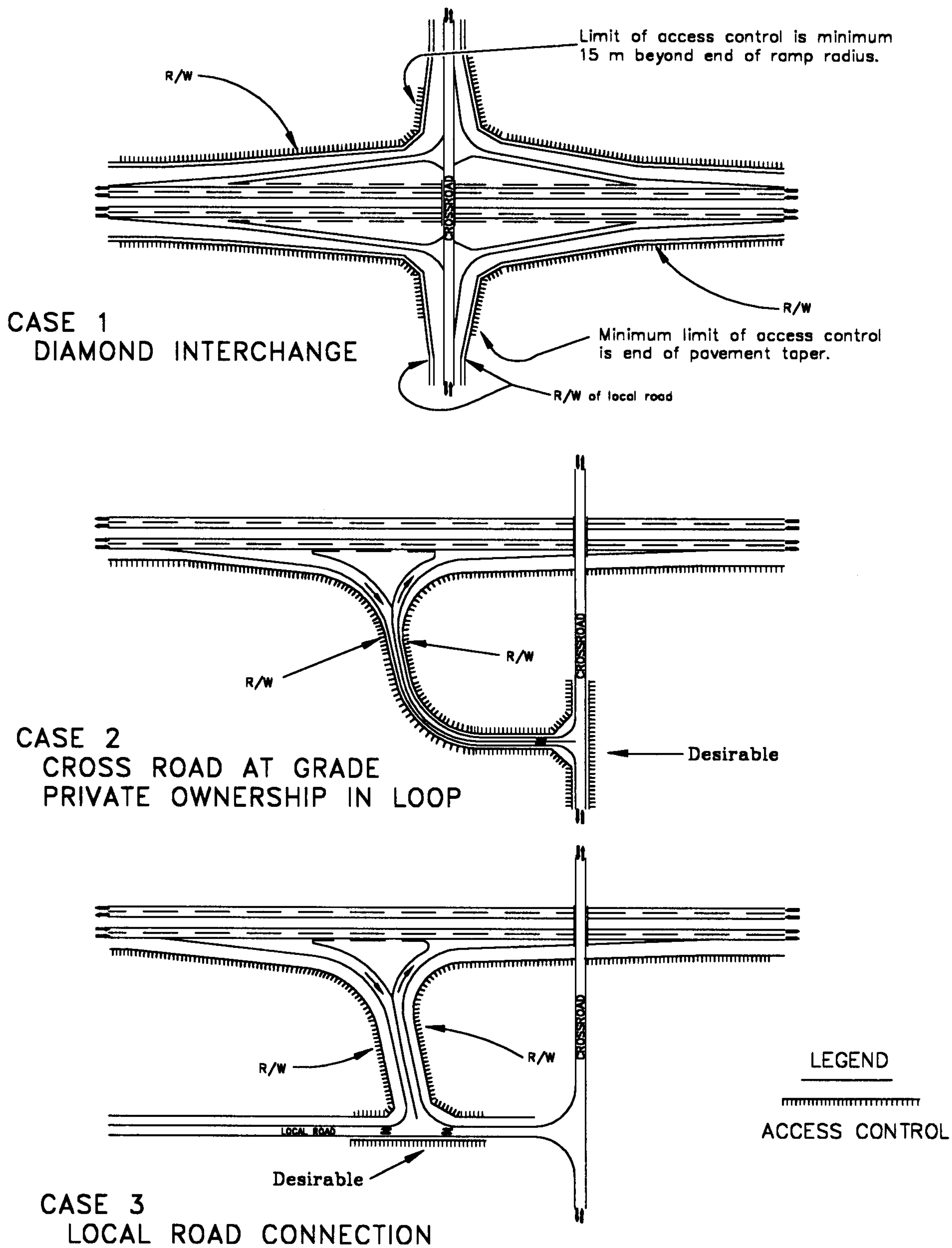
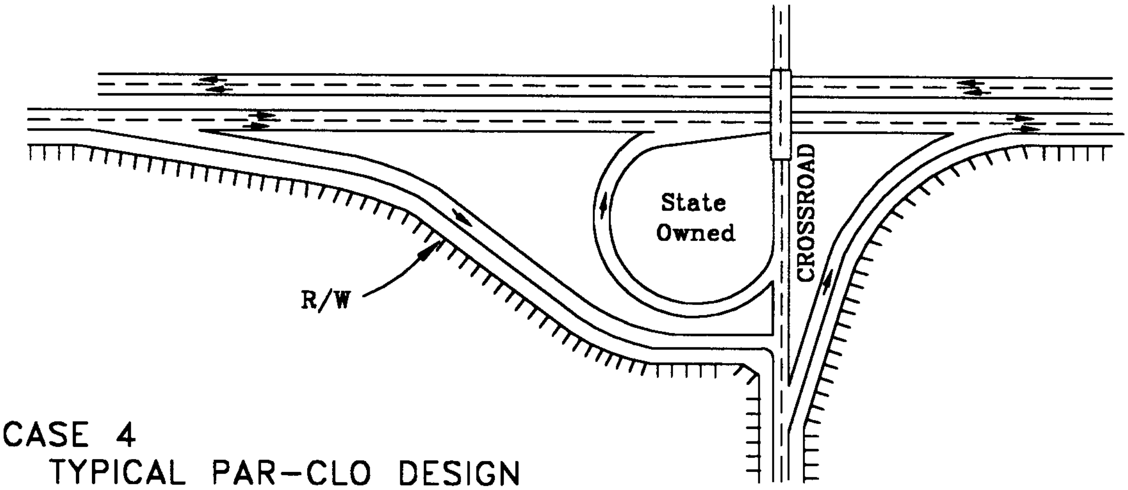
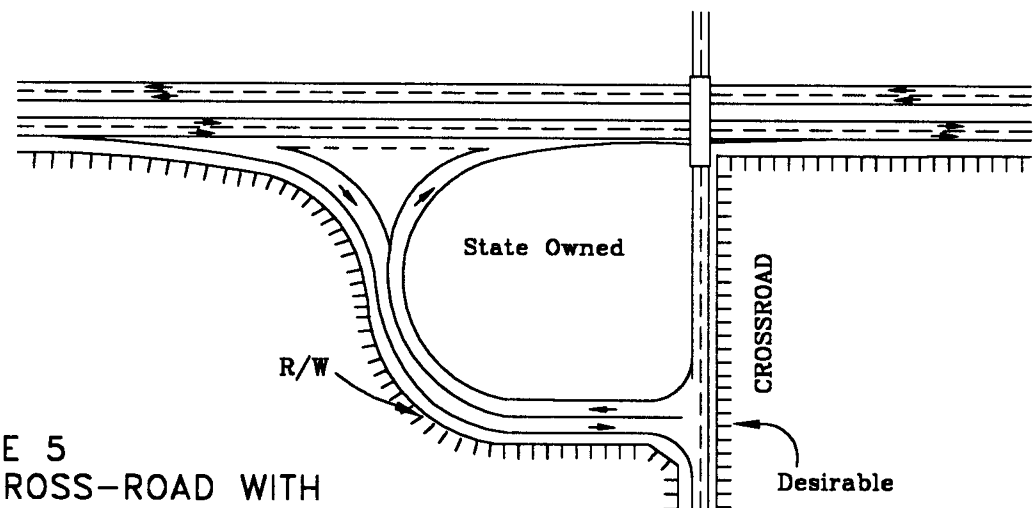


Figure 504.8 (cont.)

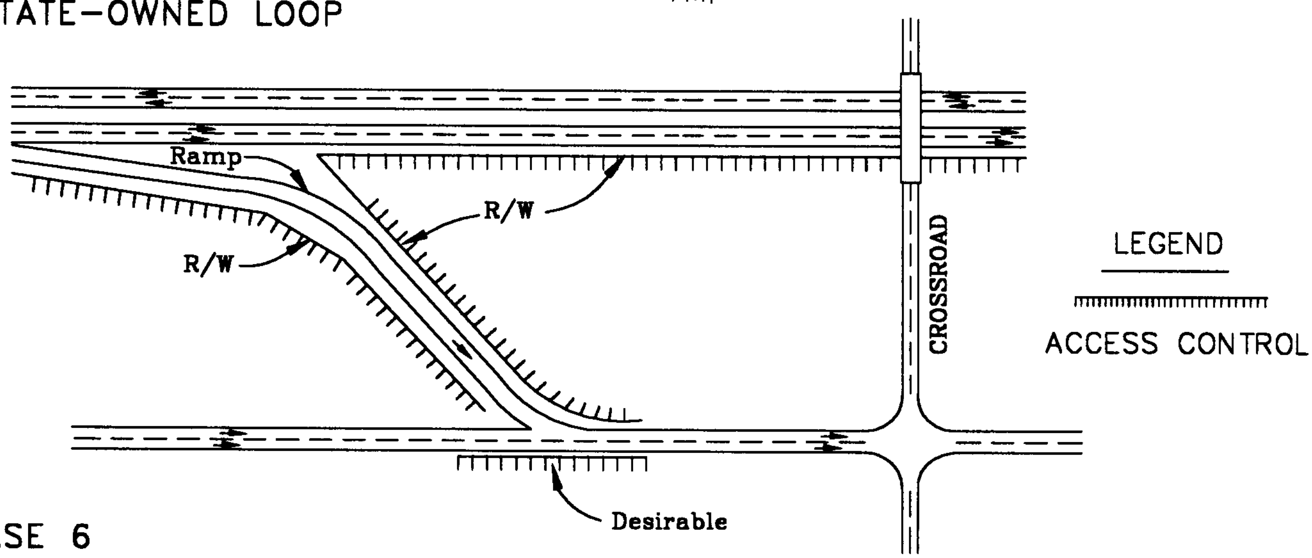
Typical Examples of Access Control at Interchanges



CASE 4
TYPICAL PAR-CLO DESIGN



CASE 5
CROSS-ROAD WITH
STATE-OWNED LOOP



CASE 6
ONE-WAY FRONTAGE ROAD